
Seismic behaviour, design and detailing of RC shear walls, Part II : Design and detailing

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Shear walls are one of the excellent means for providing earthquake resistance to multistoreyed reinforced concrete (RC) buildings. Part I of the paper published in July 1993 issue of the Journal dealt with their behaviour and strength characteristics. This part deals with specifications for the design and detailing of ductile earthquake-resistant shear walls. IS:456-1978 and IS:4326-1976 do not give specifications for the same. A detailed commentary is included to explain the basis of these specifications. A worked out example on shear wall design is also given.

Shear walls are very effective in providing lateral load resistance in multistoreyed buildings. They used to be considered as brittle structural elements. However, extensive experimentation has shown that they can be made to behave in a ductile manner by adopting proper detailing techniques. Good detailing endeavours to suppress the brittle failure modes; it enables the wall to dissipate seismic energy in the flexure mode in a stable manner. IS:456-1978 incorporates some provisions for the design of reinforced concrete walls¹. However, these provisions are inadequate for designing walls to resist earthquake induced forces. IS:4326-1976 does not have any provisions for detailing of shear walls². Building design codes

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used in the U.S.A.^{3,4,5}, Canada⁶, and New Zealand⁷ have design provisions for earthquake-resistant shear walls. However, these codes use a different design philosophy and hence cannot be directly adopted for use in India.

Part I of this paper described the behaviour and strength characteristics of shear walls⁸. This part gives recommendations for design and detailing of earthquake-resistant shear walls. An extensive study of seismic design codes used elsewhere was carried out prior to framing these provisions. The recommendations given herein are within the overall framework of IS:456-1978 and could be incorporated in IS:4326. The background for these recommendations is given through comments. One example on shear wall design and detailing is also included.

Proposed design provisions for shear walls

1. Shear walls

The requirements of this section apply to shear walls which are part of the lateral force resisting system of the structure.

1.1 General requirements

1.1.1 The thickness of any part of the wall shall not be less than 100 mm.

Comment : The minimum thickness is specified as 100 mm to avoid unusually thin sections. Very thin sections are susceptible to lateral instability in zones where inelastic cyclic loading may have to be sustained.

1.1.2 The effective flange width to be used in the design of flanged wall sections shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web; and (b) 1/10th of the total wall height.

1.1.3 Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall.

Comment : Distribution of a minimum reinforcement uniformly across the height and width of the wall helps to control the width of inclined cracks that are caused due to shear.

1.1.4 If the factored shear stress in the wall exceeds $0.25 \sqrt{f_{ck}}$ or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse direction in the plane of the wall.

Comment : The use of two curtains of reinforcement will reduce fragmentation and premature deterioration of the concrete under cyclic loading into the inelastic range. The limits of $0.25 \sqrt{f_{ck}}$ and 200 mm have been adopted from reference 7.

1.1.5 The diameter of the bars to be used in any part of the wall shall not exceed 1/10th of the thickness of that part.

Comment : This is to prevent the use of very large diameter bars in thin wall sections.

1.1.6 The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, $3 t_w$, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

1.2 Shear strength requirements

1.2.1 The nominal shear stress, τ_v , shall be calculated as

$$\tau_v = \frac{V_u}{t_w d_w}$$

where,

V_u = factored shear force

t_w = thickness of the web

d_w = effective depth of wall section. This may be taken as $0.8 l_w$ for rectangular sections.

1.2.2 The design shear strength of concrete, τ_c , shall be calculated as per Table 13 of IS:456-1978.

Comment : The vertical reinforcement that is provided in the wall shall be considered for calculation of the design shear stress of concrete as per Table 13 of IS:456-1978. The increase in shear strength due to axial compression may also be considered as per clause 39.2.2 of IS:456-1978. However, for this only 80 percent of the factored axial compressive force should

be considered as effective. This is to consider the possible effect of vertical acceleration.

1.2.3 The nominal shear stress in the wall, τ_v , shall not exceed $\tau_{c,max}$ as per Table 14 of IS:456-1978.

1.2.4 When τ_v is less than τ_c , shear reinforcement shall be provided in accordance with 1.1.3, 1.1.4, and 1.1.6 of this code.

1.2.5 When τ_v is greater than τ_c , the area of horizontal shear reinforcement, A_h , to be provided within a vertical spacing, S_v , is given by

$$V_{us} = \frac{0.87 f_y A_h d_w}{S_v}$$

where, $V_{us} = (V_u - \tau_c t_w d_w)$, is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum as per 1.1.3 of this code.

1.2.6 The vertical reinforcement that is uniformly distributed in the wall shall not be less than the horizontal reinforcement calculated as per 1.2.5.

Comment : This provision is particularly important for squat walls. When the height-to-width ratio is about 1.0, vertical and horizontal reinforcement are equally effective in resisting the shear force. However, for walls with height-to-width ratio less than 1.0, a major part of the shear force is resisted by the vertical reinforcement. Hence, adequate vertical reinforcement should be provided for such walls.

1.3 Flexural strength

1.3.1 The moment of resistance, M_{rw} , of the wall section shall be calculated as for columns subjected to combined axial load and uni-axial bending as per IS:456-1978. The moment of resistance that is provided by uniformly distributed vertical reinforcement in a slender rectangular wall section may be calculated as follows :

(a) For $x_u/l_w \leq x_u^*/l_w$

$$\frac{M_{rw}}{f_{ck} t_w l_w^2} = \phi \left[\left(1 + \frac{\lambda}{\phi} \right) \left(\frac{1}{2} - 0.416 \frac{x_u}{l_w} \right) - \left(\frac{x_u}{l_w} \right)^2 \left(0.168 + \frac{\beta^2}{3} \right) \right]$$

where,

$$\frac{x_u}{l_w} = \left(\frac{\phi + \lambda}{2\phi + 0.36} \right); \frac{x_u^*}{l_w} = \left(\frac{0.0035}{0.0035 + 0.87 \frac{f_y}{E_s}} \right)$$

$$\phi = \left(\frac{0.87 f_y \rho}{f_{ck}} \right); \lambda = \left(\frac{P_u}{f_{ck} t_w l_w} \right)$$

ρ = vertical reinforcement ratio = $A_{st} / (t_w l_w)$

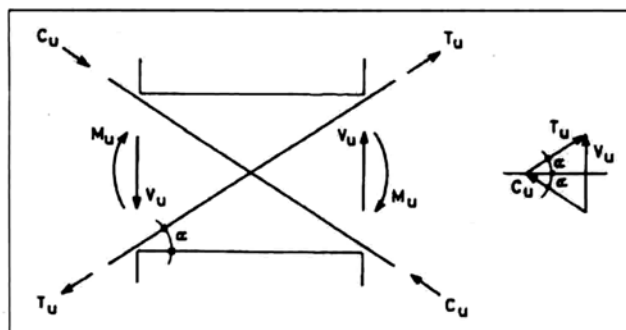


Fig 1 Diagonally reinforced coupling beam

A_{st} = area of uniformly distributed vertical reinforcement

$$\beta = 0.87 f_y / (0.0035 E_s)$$

E_s = elastic modulus of steel

P_u = axial compression on wall

(b) For $x_u'/l_w < x_u/l_w < 1.0$

$$\frac{M_{uw}}{f_{ck} t_w l_w^2} = \alpha_1 \left(\frac{x_u}{l_w} \right) - \alpha_2 \left(\frac{x_u}{l_w} \right)^2 - \alpha_3 - \frac{\lambda}{2}$$

where,

$$\alpha_1 = [0.36 + \phi (1 - \frac{\beta}{2} - \frac{1}{2\beta})]$$

$$\alpha_2 = [0.15 + \frac{\phi}{2} (1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\beta})];$$

$$\alpha_3 = \frac{\phi}{6\beta} \left[\frac{1}{(x_u/l_w)} - 3 \right]$$

The value of x_u/l_w to be used in this equation should be calculated from the quadratic equation

$$\alpha_1 \left(\frac{x_u}{l_w} \right)^2 + \alpha_4 \left(\frac{x_u}{l_w} \right) - \alpha_5 = 0$$

where,

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda \right); \text{ and } \alpha_5 = \left(\frac{\phi}{2\beta} \right)$$

Comment : These equations were derived assuming a rectangular wall section of depth l and thickness t_w that is subjected to combined uni-axial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress-strain curve assumed for concrete is as per IS:456-1978 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression.

1.3.2 The cracked flexural strength of the wall section should be greater than its uncracked flexural strength.

Comment : This provision governs those wall sections which, for architectural or other reasons, are much larger in cross section than required from strength considerations alone. Consider a wall section that is subjected to a gradual increase in moment. Initially, it is uncracked and behaves like a plain concrete section. When the cracking moment is reached, concrete in the extreme fiber ruptures in tension. A further increase in moment causes the reinforcement to take all tension on the section. Thus, the cracked flexural strength of the section should be greater than the uncracked flexural strength so as to prevent a brittle failure involving sudden fracture of the tension reinforcement.

1.3.3 In walls that do not have boundary elements, vertical reinforcement consisting of at least 4 bars of minimum 12 mm diameter arranged in 2 layers shall be provided along the edge of the wall.

Comment : Concentrated vertical reinforcement near the edges of the wall is more effective in resisting bending moment.

1.4 Boundary elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web, it is advantageous to provide them with greater thickness.

Comment : Wall sections having stiff and well confined boundary elements develop substantial flexural strength, are less susceptible to lateral buckling, and have better shear strength and ductility in comparison to plane rectangular walls not having stiff and well-confined boundary elements.

1.4.1 Where the extreme fiber compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2f_{ck}$, boundary elements shall be provided along the vertical boundaries of walls. The boundary elements may be discontinued where the calculated compressive stress becomes less than $0.15f_{ck}$. The compressive stress shall be calculated using a linearly elastic model and gross section properties.

Comment : During a severe earthquake, the flanges of a wall are subjected to high compressive and tensile stresses. Hence, the concrete needs to be well confined so as to sustain the load reversals without a large degradation in strength.

1.4.2 A boundary element shall have adequate axial load-carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force. The latter may be calculated as

$$\frac{M_u - M_{uw}}{C_w} \text{ where,}$$

M_u = factored design moment on the entire wall section

M_{uv} = moment of resistance provided by distributed vertical reinforcement across the wall section.

C_w = center to center distance between the boundary elements along the two vertical edges of the wall.

Comment : The boundary element is assumed to be effective in resisting the design moment due to earthquake induced forces along with the web of the wall. The axial compression that is required to be developed in the boundary element for this purpose is given by $(M_u - M_{uv})/C_w$. Thus, the boundary element should be designed as a short column for an axial load equal to the sum of the above axial compression and the gravity load on it.

1.4.3 If the gravity load adds to the strength of the wall, its load factor shall be taken as 0.8.

Comment : Moderate axial compression results in higher moment capacity of the wall. Hence, the beneficial effect of axial compression by gravity loads should not be fully relied upon in design due to the possible reduction in its magnitude by vertical acceleration. For example, consider a boundary element in which the unfactored axial compression due to gravity and earthquake loading is 400 kN and 50kN, respectively. Also assume that the bending moment in the wall due to seismic force exerts an additional axial force of 800 kN in the boundary element. The boundary element should be designed for the factored compression force of $1.2 [400 + 50 + 800] = 1500$ kN. Also, the factored design tension force will be $(0.8 \times 400 - 1.2 \times 50 - 1.2 \times 800) = -700$ kN. Note that a load factor of 0.8 has been used for the gravity load as the gravity load adds to the strength of the wall by reducing design tension in the boundary element.

1.4.4 The percentage of vertical reinforcement in the boundary elements shall not be less than 0.8 percent nor greater than 6 percent. In order to avoid congestion, the practical upper limit would be 4 percent.

1.4.5 Boundary elements where required as per 1.4.1, shall be provided with special confining reinforcement as is required for columns in IS:4326.

Comment : During a severe earthquake, boundary elements may be subjected to stress reversals. Hence, they have to be confined adequately to sustain the cyclic loading without a large degradation in strength.

1.4.6 Boundary elements need not be provided if the entire wall section is provided with special confining reinforcement as is required for columns in IS:4326.

1.5 Coupled shear walls

1.5.1 Coupled shear walls shall be connected by ductile coupling beams. If the earthquake induced shear stress in the coupling beam exceeds

$$\frac{0.1 l_s \sqrt{f_{ck}}}{D}$$

where, l_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake induced shear and flexure shall preferably be resisted by diagonal reinforcement.

Comment : Coupling beams must have large ductility as they are subjected to extensive inelastic deformations at their ends. In coupling beams of small span-to-depth ratio, diagonal reinforcement is much more effective in controlling shear displacements and in preventing sliding shear failure as compared to conventional parallel reinforcement. The limit on shear stress has been adopted from reference 7.

1.5.2 The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be

$$A_{sd} = \frac{V_u}{1.74 f_y \sin \alpha}$$

where, V_u is the factored shear force, and α is the angle made by the diagonal reinforcement with the horizontal. At least 4 bars of 8 mm diameter shall be provided along each diagonal. The reinforcement along each diagonal shall be enclosed by special confining reinforcement as is required for columns in IS:4326. The pitch of spiral or spacing of ties shall not exceed 100 mm.

Comment : The design of a diagonally reinforced coupling beam is based on the assumption that the shear force resolves itself into diagonal compression and tension forces, Fig 1. These forces intersect each other at midspan where no moment is to be resisted. Thus, the shear force will be equal to $(2 T_u \sin \alpha)$, where $T_u = 0.87 f_y A_{sd}$. The diagonal bars that are in compression need to be restrained against buckling. Hence, special confining reinforcement has to be provided all along their length.

1.5.3 The diagonal or horizontal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension.

Comment : This increase in development length is to consider the adverse effect of reversed cyclic loading on the anchorage of a group of bars.

1.6 Openings in walls

1.6.1 The shear strength of a wall containing openings should be checked along critical planes that pass through openings.

Comment : An opening in a shear wall causes high shear stresses in the region of the wall adjacent to it. Hence, it is necessary to check such regions for adequacy of horizontal shear reinforcement in order to prevent a diagonal tension failure due to shear.

1.6.2 Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars. The vertical bars should extend for the full story height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

1.7 Discontinuous walls

Columns supporting discontinuous walls shall be provided with special confining reinforcement as per IS:4326 over their full height. The column reinforcement shall be extended into the wall for a distance equal to the development length of the largest longitudinal bar in the column.

Comment : Columns supporting discontinued shear walls may be subjected to significant axial compression and may have to undergo extensive inelastic deformations. Hence, they have to be adequately confined over their full height to ensure good ductility.

1.8 Construction joints

The vertical reinforcement ratio across a horizontal construction joint shall not be less than

$$\frac{0.92}{f_y} \left(\tau_v - \frac{P_u}{A_g} \right)$$

where, τ_v is the factored shear stress at the joint, P_u is the factored axial force (positive for compression), and A_g is the gross cross sectional area of the joint.

Comment : The design shear force at the joint must be less than the shear force that can be safely transferred across the joint, V_j . This is calculated by the shear friction concept and is given by

$$V_j = \mu (0.8 P_u + 0.87 f_y A_v)$$

where, μ is the coefficient of friction at the joint ($\mu = 1.0$), and A_v is the area of vertical reinforcement available. To account for the possible effects of vertical acceleration, the axial load is taken as $0.8 P_u$ instead of P_u itself.

1.9 Development, splice and anchorage requirement

1.9.1 Horizontal reinforcement shall be anchored near the edges of the wall or in the confined core of the boundary elements.

Comment : Horizontal reinforcement acts as web reinforcement for resisting the shear force. Hence, it should be well anchored.

1.9.2 Splicing of vertical flexural reinforcement should be avoided as far as possible in regions where yielding may take place. This zone of flexural yielding may be considered to extend for a distance of l_w above the base of the wall or one sixth of the wall height, whichever is more. However, this distance need not be greater than $2l_w$. Not more than one third of this vertical reinforcement shall be spliced at such a section. Splices in adjacent bars should be staggered a minimum of 600 mm.

1.9.3 Lateral ties shall be provided around lapped spliced bars that are larger than 16 mm in diameter. The diameter of the tie shall not be less than one fourth that of the spliced bar nor

less than 5 mm. The spacing of ties shall not exceed 150 mm on centres.

1.9.4 Welded splices and mechanical connections shall conform to clause 25.2.5.2 of IS:456-1978. However, not more than half the reinforcement shall be spliced at a section where flexural yielding may take place.

A worked out example is given in the *Appendix*.

Conclusions

Shear walls are very suitable for resisting earthquake induced lateral forces in multistoreyed building systems. They can be made to behave in a ductile manner by adopting proper detailing techniques. IS:456-1978 and IS:4326-1976 do not give specifications for them. Hence, provisions are proposed for the same. These provisions endeavour to suppress all brittle failure modes and promote seismic energy dissipation in the flexure mode. Provisions are given for shear and flexure design of walls, for design and detailing of the wall web, boundary elements, wall openings, construction joints, and development, splice, and anchorage requirement for reinforcement. A commentary is included to explain the basis of the provisions. An example on shear wall design is also given to illustrate application of the intended provisions.

Acknowledgment

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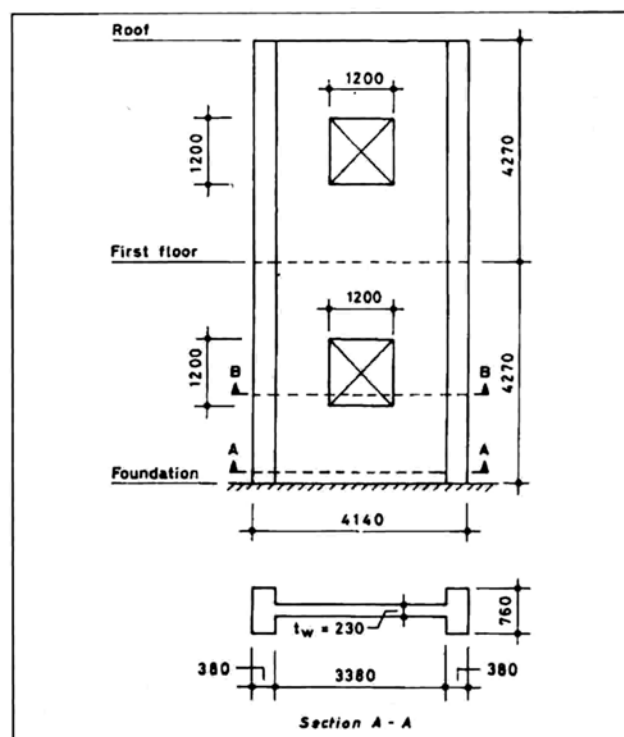


Fig 2 shear wall details for case illustrated in the solved example

Appendix : Example

A shear wall for a two-storey building, Fig 2, has been designed and detailed in this section. The materials are M15 concrete and Fe 415 steel. The unfactored forces in the panel between the ground level and first floor are obtained by analysis as

Sr.no	Load case	Bending moment kNm	Axial force kN	Shear force kN
1.	(DL + LL)	-577.5	1922.9	19.7
2.	Earthquake	4830.9	255.7	699.1

The maximum factored bending moment on the section = $1.2 \times (577.5 + 4830.9) = 6490$ kNm. The maximum factored shear force = $1.2 \times (19.7 + 699.1) = 863$ kN.

(a) **Shear design** : At section A-A, the design shear force is, $V_u = 863$ kN. Let the effective depth in resisting shear be 3760 mm (3380 + 380). Therefore, $\tau_v = 0.998$ N/mm². Let minimum vertical reinforcement (0.25 percent) be provided in the web. Therefore, as per Table 13 of IS:456-1978, $\tau_c = 0.350$ N/mm². Hence, shear to be resisted by horizontal reinforcement is, $V_{us} = 560$ kN. This requires the ratio A_h/S_v to be 0.413. However, provision of minimum horizontal reinforcement (0.25 percent) requires this ratio to be 0.575. As $t_w > 200$ mm, the reinforcement shall be in 2 layers. Thus, horizontal reinforcement of 8 mm diameter bars at 175 mm c/c in 2 layers will suffice. An opening is present at section B-B. Taking depth of wall on each side of opening that is resisting shear as 1280 mm, $\tau_v = 1.466$ N/mm². Thus, shear to be resisted by reinforcement on each side of opening is, $V_{us} = 328$ kN. Therefore, provide 8 mm diameter 2-legged stirrups at 140 mm c/c on each side of opening.

(b) **Flexural strength of web** : The vertical reinforcement in the web is 0.25 percent. The length of the wall, l_w , is 4140 mm and its web thickness, t_w , is 230 mm. Axial compression will increase the moment capacity of the wall. Therefore, the factored axial force should be taken as $P_u = 0.8 \times 1922.9 + 1.2 \times 255.7 = 1845$ kN. Assuming this axial load to be uniformly distributed, load on web = $0.574 \times 1845 = 1059$ kN. Thus, from equations in 1.3.1 (a) for $x_u/l_w \leq x_u^*/l_w$, $\lambda = 0.074$, $\phi = 0.060$, $x_u/l_w = 0.279$, $x_u^*/l_w = 0.660$, and the value of β is 0.516. As x_u/l_w is less than x_u^*/l_w , the moment of resistance of the web is obtained from equations in 1.3.1 (a) as, $M_{wv} = 2967$ kNm. The remaining moment, that is, $M_u - M_{wv}$, shall be resisted by reinforcement in the boundary elements.

(c) **Boundary elements** : The axial compression at the extreme fiber due to combined axial load and bending on the section is 6.805 N/mm². As this is greater than $0.2 f_{ck}$, provision of boundary elements along the wall edges is mandatory. The center to center distance between the boundary elements, C_w , is 3.760 m. The axial force on the boundary element due to earthquake loading is $(M_u - M_{wv})/C_w = 937$ kN. Thus, the maximum factored compression on the boundary element is $[937 + 0.213 \times 1.2 \times (1922.9 + 255.7)] = 1494$ kN. The factored tension on the boundary element is $[0.213 \times (0.8 \times 1922.9 - 1.2 \times 255.7) - 937] = -675$ kN. Assuming short column action, the axial load capacity of the boundary element with minimum reinforcement of 0.8 percent is 2361 kN. Therefore, 12 bars of 16 mm diameter will be adequate to take the compression as well as tension. The arrangement

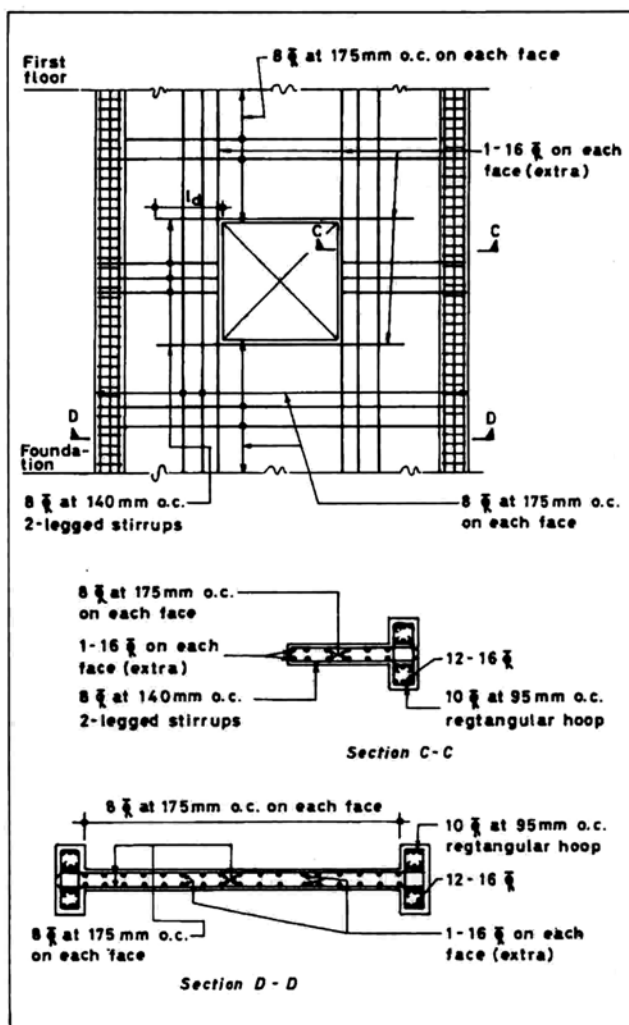


Fig 3 Reinforcement details for example wall

of reinforcement in the boundary element as per Fig 3 requires 10 mm diameter rectangular hoops to be provided at 95 mm on centres as special confining reinforcement.

(d) **Reinforcement around opening** : The opening is of size 1200 mm by 1200 mm. The area of vertical and horizontal reinforcement in the web (0.25 percent) that is interrupted by it is 690 mm². Therefore, 1 bar of 16 mm diameter should be provided per layer of reinforcement on each side of the opening. The vertical bar should extend for the full storey height. The horizontal bar should be provided with development length in tension beyond the sides of the opening.

Fig 3 illustrates the reinforcement details.

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New Publication from ACC-RCD

Concrete Mix Design

"Concrete Mix Design" which was first brought out by the erstwhile Concrete Association of India in 1979 and then in 1983 was found extremely useful by practicing engineers, consultants, concrete technologists and others connected with the concrete construction industry. This booklet has been recently revised and an enlarged edition is published by the Research and Consultancy Directorate of the Associated Cement Cos Ltd.

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